

LATERAL LOADED PILE/SHAFT RESPONSE IN LIQUEFIED SOIL AND ANTICIPATED LATERAL SOIL SPREADING

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ABSTRACT

This paper provides a new analysis procedure for assessing the lateral response of an isolated pile in saturated sands as liquefaction develops in response to dynamic loading such as that generated during earthquake shaking. This new procedure predicts the degradation in pile response and soil resistance due to the free-field excess porewater pressure, generated by the earthquake, along with the near-field excess porewater pressure generated by lateral loading from the superstructure. The new procedure involves the integration of the developing (free- and near-field) porewater pressure in the strain wedge (SW) model analysis. At present, the SW model is the only technique that allows the assessment of the p-y curve in liquefied soil considering partially and fully liquefied soil scenario. The full-scale load tests at Treasure Island have proven and showed the capability and efficiency of the SW model compared to other available approaches. It should be noted that the findings of full- and model-scale tests sponsored by Caltrans should be compiled and implemented in a reliable and user-friendly design tool such as the SW model technique. The analytical and empirical concepts employed in the SW model technique allow the extension of this technique to handle the sophisticated phenomenon of the lateral soil spreading that could accompany or follow the occurrence of seismic events. As a result, the p-y curve of liquefied soil with lateral spreading can be assessed based on soil and pile properties and the characteristics of the seismic event. The amount of soil lateral spreading can also be calculated to provide a representative p-y curve (i.e. a realistic pile/shaft lateral response) without using modifying parameters or shape corrections. This will be visible in our next phase of research with Caltrans.

INTRODUCTION

The procedure presented predicts the post-liquefaction behavior of laterally loaded piles in sand under developing or fully liquefied conditions. Due to the shaking from the earthquake and the associated lateral load from the superstructure, the free field $u_{ks,ff}$ and near-field $u_{ks,nf}$ develop and reduce the strength of loose to medium dense sand around a pile. The soil is considered partially liquefied or experiencing developing liquefaction if the excess porewater pressure ratio (r_u) induced by the earthquake shaking (i.e. $u_{ks,ff}$) is less than 1, and fully liquefied if $r_u = 1$.

Therefore, the stress-strain response of the soil due to the lateral push from the pile as the result of superstructure load (and $u_{ks,nf}$) can be as shown in Fig. 1. The full-scale load tests on the post-liquefaction response of isolated piles and a pile group, performed at Treasure Island (Rollins et al. 2005 and Weaver et al. 2005) are the most significant related tests. However, the profession still lacks a realistic procedure for the design of pile foundations in liquefying or liquefied soil.

The most common practice employed is that presented by Wang and Reese (1998) in which the traditional p-y curve for clay is used but based on the undrained residual strength (S_r) of the sand. As seen in Fig. 2 (Seed and Harder 1990), S_r can be related to the standard penetration test (SPT) corrected blowcount, $(N_1)_{60}$. However, a very large difference between values at the upper and lower limits at a particular $(N_1)_{60}$ value affects the assessment of S_r tremendously. Even if an accurate value of S_r is available, S_r occurs at a large value of soil strain. In addition, a higher peak of undrained resistance is ignored in the case of the partially liquefied sand, while greater resistance at lower strain is attributed to the sand in the case of complete liquefaction. Such clay-type modeling can, therefore, be either too conservative (if $r_u < 1$) or unsafe (if $r_u = 1$). Furthermore, the p-y curve reflects soil-pile-interaction, not just soil behavior. Therefore, the effect of soil liquefaction (i.e. degradation in soil resistance) does not reflect a one-to-one change in soil-pile or p-y curve response.

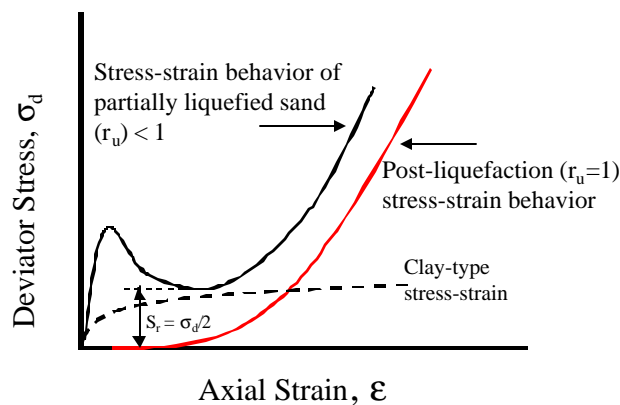


Fig. 1. Subsequent Undrained Stress-Strain Behavior of Partially ($r_u < 1$) and Fully (1990). Liquefied Soil.

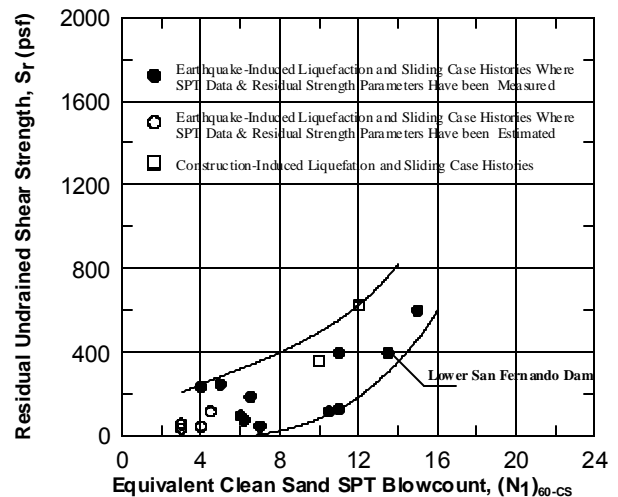
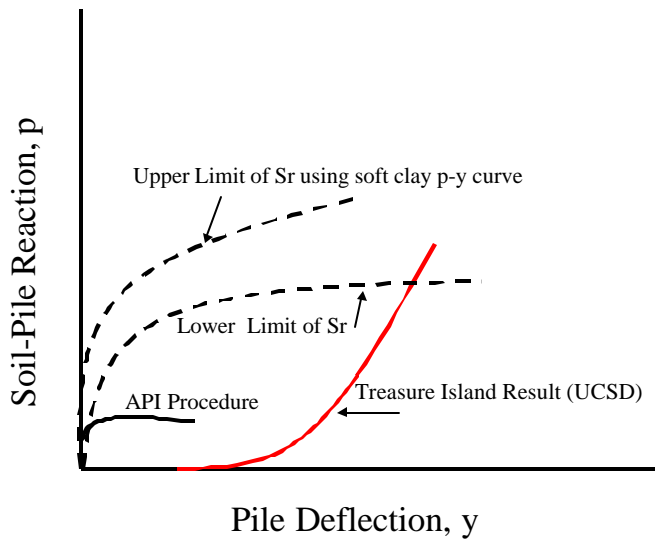


Fig. 2. Corrected Blowcount vs. Residual of Strength (Seed and Harder

SOIL LIQUEFACTION

The post-liquefaction stress-strain characterization of a fully or partially liquefied soil is still under investigation by several researchers. The current assessment of the resistance of a



liquefied soil carries a lot of uncertainty.

With lateral loading from the superstructure following full liquefaction or partial liquefaction with a significant drop in the confining pressure, the sand responds in a dilative fashion. However, a partially liquefied sand with small drop in confining pressure may experience contractive behavior followed by dilative behavior under a compressive monotonic loading. The postcyclic response of sand, particularly after full liquefaction, reflects a stiffening response, regardless of its initial (static) conditions (density or confining pressure).

Fig.3. Undrained p-y Curve In Liquefied Soil And Fully Liquefied (Rollins et al. 2001)

As seen in Fig. 3, there is no particular technique that allows the assessment of the p-y curve and its varying pattern in a partially or fully liquefied sand. Instead, the soil's undrained stress-strain relationship should be used in a true soil-pile interaction model to assess the corresponding p-y curve behavior. Because the traditional p-y curve is based on field data, a very large number of field tests for different pile types in liquefying sand would be required to develop a realistic, empirically based, p-y characterization.

The existing technique concerns the degradation in soil resistance due to earthquake shaking and the induced porewater pressure in the free-field ($u_{ks,ff}$) is based on the procedures proposed by Seed et al. 1983. This $u_{ks,ff}$ reduces the effective stress of the soil. Thereafter, the lateral load (from the superstructure) is applied at the pile head that generates additional porewater pressure ($u_{ks,nf}$) in the near-field soil immediately around the pile causing an additional degradation in soil strength already reduced by $u_{ks,ff}$. Note that $u_{ks,ff}$ is taken to reduce the vertical effective stress from its pre-earthquake state \bar{s}_{vo} to $\bar{s}_{vo}(1-r_u)$. Thereafter, the undrained behavior due to an inertial induced lateral load is assessed using undrained stress-strain formulation (Ashour and Norris 1999 and Ashour 2002) in the extended SW model. This procedure incorporates the whole undrained stress-strain curve (at any level of loading) not only the residual strength of the sand as presented by Ashour and Norris (2003).

ANTICIPATED SOIL LATERAL SPREADING

The major challenges in the analysis of piles/shafts in liquefied soil undergoing lateral spreading are **1)** how far the crust layer would move; **2)** the undrained behavior (varying strength) of the liquefied soil layer in the near-field; and **3)** the amount of driving (inertial) force on the piles. The technique suggested allows the assessment of the undrained stress-strain-strength relationship of a fully ($r_u = 1$) or partially ($r_u < 1$) liquefied soil as seen in Fig. 1. Therefore, the mobilized strength of the liquefied soil can be assessed according to level of soil strain. The

lateral soil spreading analysis implemented assumes that the crust layer keeps applying increasing lateral driving force on the piles as long as the underlying soil layer(s) is fully liquefied (Phase I in Fig. 4). Once the fully liquefied soil layer starts gaining some strength (i.e. $r_u < 1$) due to progressive deformation, the overlying crust layer switches from applying driving force to providing passive resistance to the pile lateral deflection (Phase 2 in Fig. 4).

Figure 4 shows the modeling (characterization) of a pile in liquefied soil undergoing spreading and the shape of the associating p-y curves in the liquefied and nonliquefied soil layers. The suggested technique allows the evaluation of the lateral displacement of the liquefied soil (Δy_s) [i.e. the associated displacement of the upper nonliquefied soil(s), Δy_{sl}] before the shear strength of the liquefied soil starts picking up (rebound). In addition, the varying driving force exerted by the crust on the pile during the lateral spreading (phase I) can be determined based on the interaction between the pile and the surrounding soil. Therefore, the resulting p-y curve in the crust will account for the displacement caused by the lateral spreading of the underlying soils as seen in Fig. 4. In addition, the p-y assessed for the liquefied soil will account for the varying strength of the soil and the continuous changes in the water pressure at any level of loading.

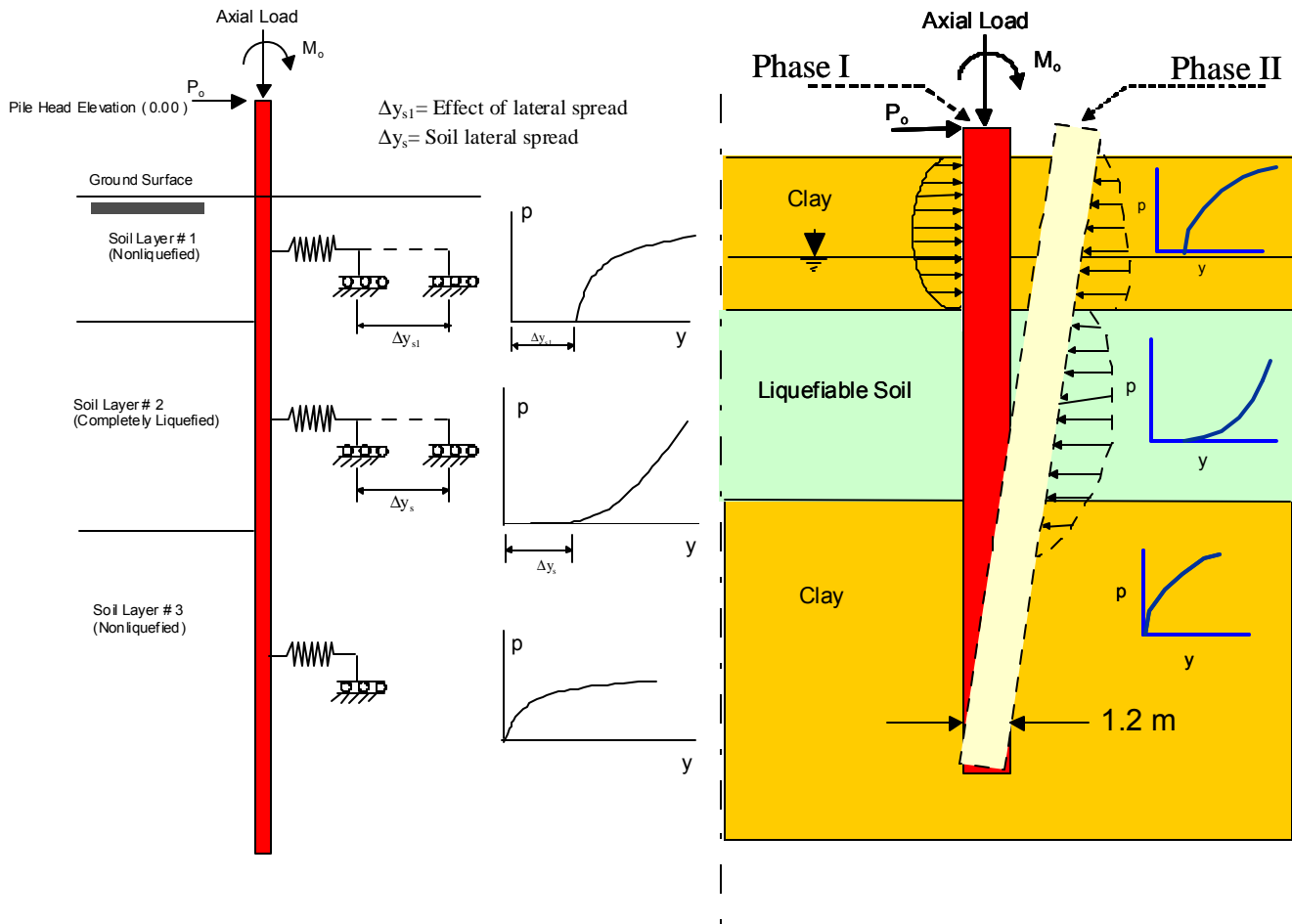


Fig. 4 The Mechanism of a Pile/Shaft in Liquefied Soil Undergoing Lateral Spreading

Treasure Island Full-Scale Load Test of Isolated Pile (0.61 m CISS Pile) in Liquefied Soil

Table 1. Soil Properties Employed with Treasure Island Test (Weaver et al. 2005)

Soil Layer Thick. (m)	Soil Type	Unit Weight, $\bar{\gamma}$ (kN/m ³)	(N ₁) ₆₀	F (degree)	ϕ_{50} %	*S _u kN/m ²
0.5	Brown, loose sand (SP)	18.0	16	33	0.45	
4.0	Brown, loose sand (SP)	8.0	11	31	0.6	
3.7	Gray clay (CL)	7.0	4		1.5	20
4.5	Gray, loose sand (SP)	7.0	5	28	1.0	
5.5	Gray clay (CL)	7.0	4		1.5	20

* Undrained shear strength

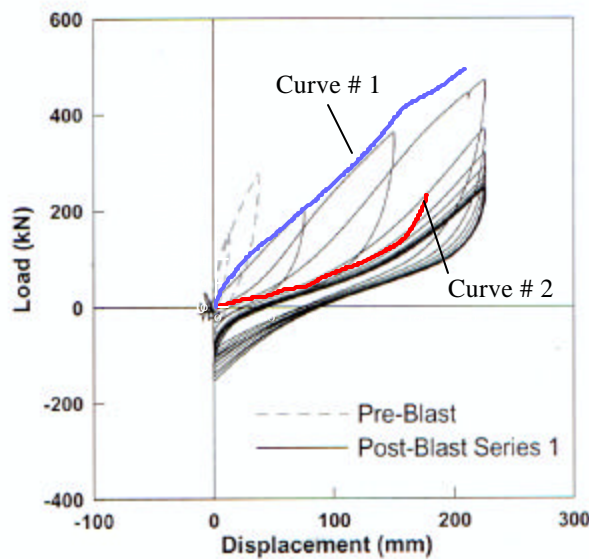


Fig. 5 Post-Liquefaction Pile-Head Response at Treasure Island Test (0.61 m).

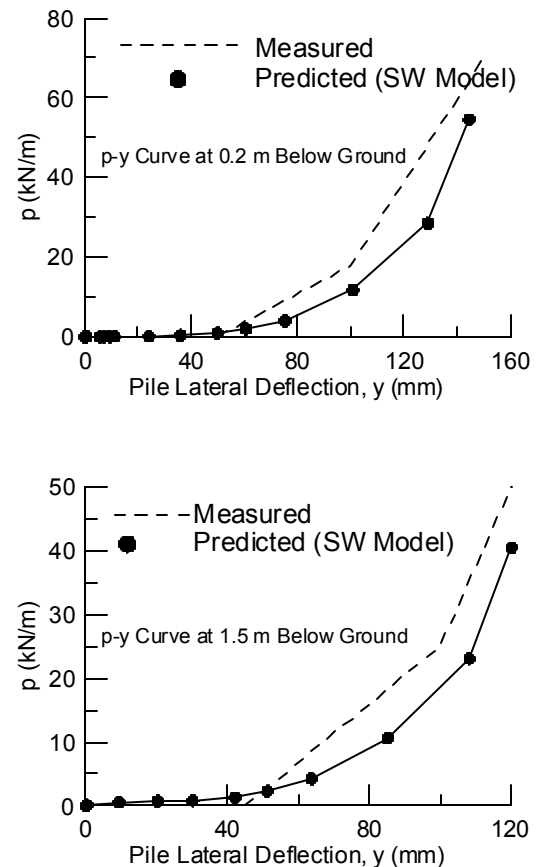


Fig. 6 Computed p-y Curves vs. Observed Ones for Treasure Island Test (0.61-m-CISS).

A full-scale load test of the post-liquefaction response of long isolated piles performed at the Treasure Island site (Weaver et al. 2005) provides the data with which to evaluate the capability

of SW model (Ashour et. 1998 and Ashour and Norris 2003) to predict laterally loaded pile response in liquefied soil. The soil properties employed in the SW model analysis for the test site and provided in Table 1 are based on the data reported by Weaver et al. (2005). The sand is assumed to contain 10% fines. The soil was liquefied by carrying out controlled blasts at that site without densifying the soil in the test area. Drained and undrained lateral loading tests were performed on an isolated CISS (cast in steel shell) pile of 0.61 m diameter. The tested pile exhibited free-head conditions and was laterally loaded 1.0 m above ground surface.

The predicted and observed drained response of the tested pile compare favorably as seen in Fig. 5. The assessed post-liquefaction undrained behavior of the tested pile is based on the consideration of the effect of porewater pressure in the free- and near-field. The pile was cyclically loaded after the first blast at the site. It should be mentioned that the good agreement between the measured and predicted undrained response Fig. 5 is based on a peak ground acceleration, a_{max} , of 0.1g and an earthquake of magnitude 6.5. This value of a_{max} generates high porewater pressures ($u_{ks, ff}$) of $r_u \approx 0.9$ in most of the upper sand layers and the best match with the measured free-field porewater pressure pattern induced in the field.

The computed results represented by curve # 1 in Fig. 5 are in good agreement with the data collected during the first 4 cycles of loading. However, by the seventh cycle of loading, the rising water generated by liquefaction covered the ground surface and as a result the pile-head load response took a concave-up pattern due to progressive soil liquefaction around the pile (curve # 2 in Fig. 5). The computed pile-head response takes the shape of curve # 2 based on the updated soil profile. As mentioned by Weaver et al. (2005) and Rollins et al. (2005), the concave-up p-y curves shown in Fig. 6 were back-calculated at the seventh cycle of loading. Excellent agreement between computed and backcalculated p-y curve can be seen in Fig. 6.

Treasure Island Full-Scale Load Test on a Long Isolated Pile and Pile Group (0.324-m CISS Pile) in Liquefied Soil Profile

Table 2 Soil Properties Employed in the Analysis for Treasure Island Test (Rollins et al. 2005)

Soil Layer Thick. (m)	Soil Type	Unit Weight, $\bar{\gamma}$ (kN/m ³)	(N ₁) ₆₀	F (degree)	ϕ_{50} %	*S _u kN/m ²
0.5	Sand	19.5	16	38	0.4	
2.5	Sand	10.3	12	38	0.4	
1	Sand	10.3	10	36	0.5	
2	Sand	10.3	6	33	0.8	
1.5	Sand	10.3	7	34	0.7	
1.75	Clay	9.5	3	0	2.0	20
1	Sand	10.3	8	33	0.8	
1.7	Clay	9.5	3	0	2	20

The 3 x 3 steel pipe pile group (0.324 m diameter) at the Treasure Island was tested in the soil profile exhibiting drained behavior. The group was retested in the soil liquefied by controlled blasting as addressed by Rollins et al. (2005). An earthquake event with a magnitude of 6.5 and

a peak ground acceleration (a_{max}) of 0.1 g was employed in SW model analysis. The soil profile presented in Table 2 (Rollins et al. 2005). was used in the SW model analysis. The field data shown in Fig. 7 for the isolated pile and pile group were computed for the first 4 cycles of loading (curve #1) and at the seventh cycle of loading (curve #2) when the soil profile reached peak liquefaction conditions and the rising water generated by liquefaction covered the ground surface (the upper soil layer has become below the water table).

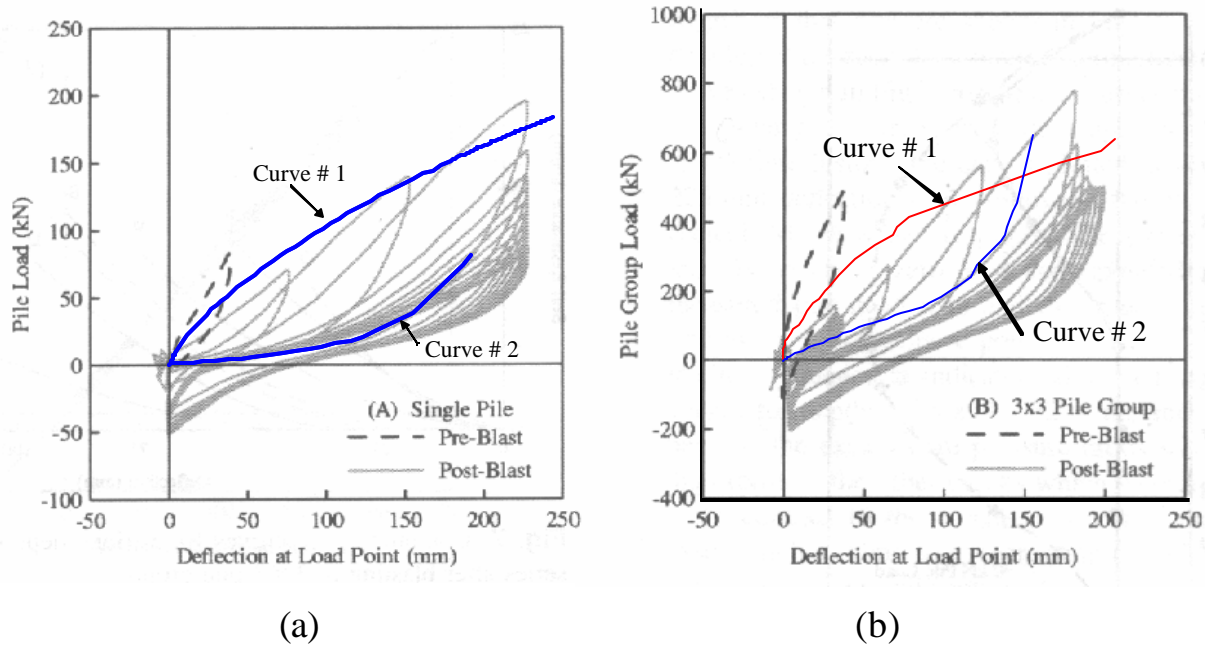


Fig. 7. Computed vs. Measured Response of 0.324 m Isolated Pile (a) and 3 x 3 Pile Group (b) in Liquefied Soil of the Treasure Island Test (after Rollins 2005)

The assessed post-liquefaction behavior of the pile group is based on the consideration of the effect of porewater pressure in the free- and near-field zones, and group interaction. The SW model calculated p-y curves shown in Figs. 8 and 9 for isolated pile and pile group are in good agreement with the back-calculated ones presented by Rollins et al. (2005).. It should be noted that because of fully liquefaction conditions, the group effect on the p-y curves shown in Figs. 8 and 9 is very limited. However, partial liquefaction conditions will exhibit greater group effect at the same depths.

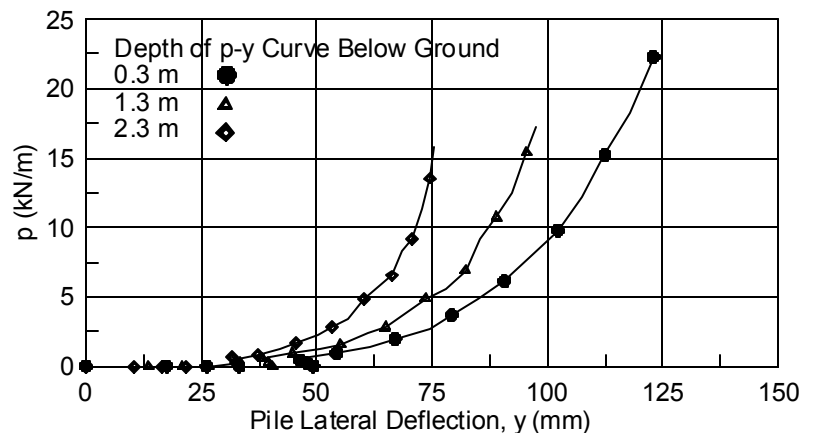
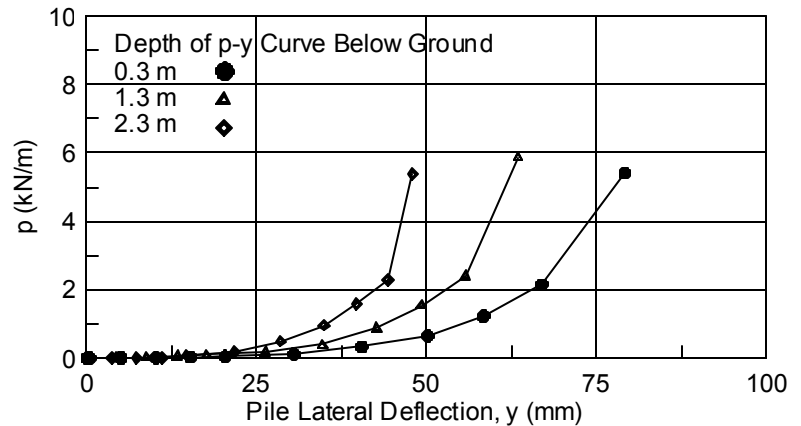


Fig. 8 P-Y Curve for the Single 0.324-m-CISS Pile under Liquefaction Conditions at Different Depths (Treasure Island Test).

Fig. 9 P-Y Curve for Individual 0.324-m-CISS Pile in a 3 x 3 Pile Group Pile under Liquefaction Conditions at Different Depths (Treasure Island Test)



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